CONSOLIDATION OF CATHEDRAL OF PORTO

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Abstract

The present paper presents the works recently carried out at the Cathedral of Porto as a case study of a difficult intervention that challenges current recommendations for the architectural heritage. The aspects regarding structural consolidation are addressed, including strengthening and monitoring of the towers and diagnostics of a chapel.

Introduction

A constellation of professions has been mobilized and the dialogue has been applied as a methodology to carry out the works in the works recently carried out at the Cathedral of Porto, which are only superficially from an author and much more from the collection of professionals.

Preliminary diagnosis was very brief and, therefore, the design project was not conceived in detail. On the contrary, the project was a directive kept open and continuously adapting to the unforeseen, which is often the case in historical buildings, where the anatomy is processed by successive approximations and under the lens of different disciplines. Of course, dissection as a knowledge tool cannot be a part of a modern intervention.

In this case, the lack of adequate preliminary diagnosis, which is in opposition with modern methodologies, was compensated with an intense multidisciplinary activity during the execution period (2002-2006), supported by research, consultancy and expertises in various fields. In the contingency of works that had started already, the reunion of efforts resulted in a process of effective cooperation, with the advantage of permanent in situ approaches and discussions.

The restoration carried out in the first half of the 20th century used traditional construction techniques. Some of the structural deficiencies encountered were then solved with the dismantling and rebuilding of unstable parts, and with the replacement of deteriorated or damaged granite, with poor mechanical performance. The sole concession to the industrial technology is the use of Portland cement, used as a common binder for repointing masonry joints, rendering walls and several reparations that during and after the restoration works, aimed at solving the following is-sues, without success: waterproofing of surfaces, glue and reconstitute volumes, stabilize cracks and stop movements. It is precisely with respect to the above-cited issues that deeper interventions have currently been carried out, some without visible effects and other with the addition of parts, as in the strengthening of the towers. Therefore, the architects in charge of the works tend to joke about the fact that the only intervention carried out was the strengthening of the towers. Next, the structural parts of the intervention are addressed. Other aspects of the intervention are addressed in [1].

Intervention of the towers

Introduction

A key aspect in the behaviour of ancient towers is that the collapse process usually excludes the possibility of ductile behaviour. In fact, there are hardly any possibilities of internal force redistributions between different critical sections, and failure of a single section is usually sufficient to provoke the entire collapse of the structure. This intrinsic feature leads to a high structural risk in tall masonry towers, because increasing height means large vertical loads and high compressive stresses at the base. Therefore, it seems easy to accept that masonry towers should possess a higher safety margin than the values normally found for other historical structures. But this is not often the case [2].

A tower is usually a result of the need to create a symbol or the need to challenge structural stability (and nature itself). The interpretation of this desire to build higher, and simultaneously to reduce the safety of structures, was left to ancient builders in the context of almost no scientific basis. It is striking that the majority of the ancient high towers in Italy, e.g. in Pavia and Bologna, are no longer present [3]. The reality is that only a few of these structures survived until today, due to collapses, destruction due to lightening and even demolitions (often by precaution and concern of eminent collapse).

Description

The main façade was built between 1176-1200 (central part) and 1229-1325 (towers), see Figure 1. The towers evolved into a Bell-tower (North) and a Clock-tower (South). In 1552, damage due to lightening is reported in the South tower. Between 1665-1669 the South tower was demolished up to mid-height and rebuilt. In 1717, it is recorded that the South tower was in the verge of collapse and, in 1727, buttresses were added, similarly to the ones that already existed in the North tower. Pinnacles were added in 1732. The construction of the Chapter House, contiguous to the South tower, also aimed at consolidating the tower. Also in this period, the two small windows in the main façade (South tower) were replaced by a single large window, similar to the one that existed in the North tower. Before 1841, a new lightening stroke the South tower.

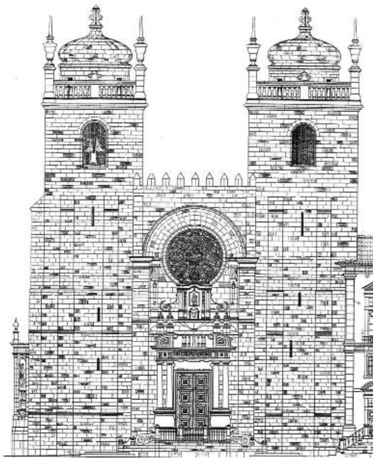
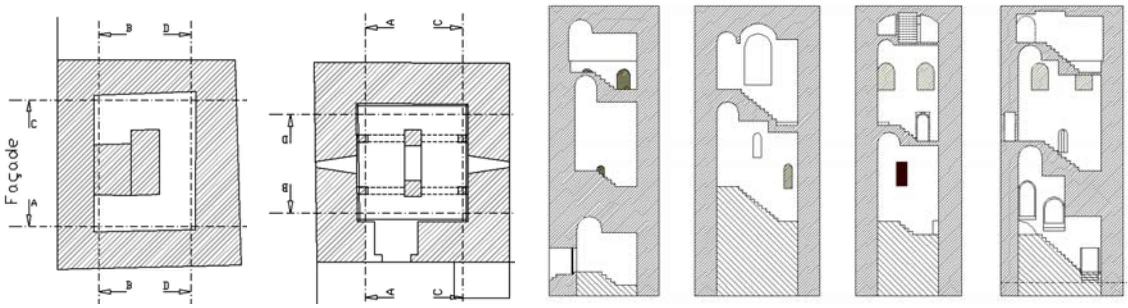


Figure 1. Main façade after the restoration works in the first half of the 20th century.

The cross section of the towers is approximately square with a side of 10.0 m and exhibits a variable thickness, with a minimum of 1.7 m at the base. The height of the towers is approximately 35 m, which means that the average stress at the base is around 1.0 N/mm^2. This value is rather low for regular granite masonry but it is rather high for rubble masonry (with or without mortar joints). In the main façade, two buttresses are apparent in each tower, see Figure 1. As addressed above, the structure suffered several major modifications through time, which resulted in a very complex internal structure with different load bearing internal elements at each level. The structure of the towers cannot be understood from structural reasons and several openings are closed, facing staircases or vaults. The entrance for both towers is located at mid-height, with a connection between both towers from the top of the main vault. But the two towers have a rather different structure. The South tower possesses an internal core with a staircase shaped helicoidally, see Figure 2a,c. The North tower (presently with the bells and clock) features a horizontal mid-level with stone slabs and architraves apparently supported in columns and stone struts, see Figure 2b.



(a) (b) (c)

Figure 2. Partial sections of the towers: (a) horizontal section of the South tower, (b) horizontal section of the North tower and (c) sections A-D for the South tower.

Constitution of masonry walls

The constitution of the masonry walls from the towers was characterized using visual inspection, both by removing smaller stones of the outer leaves and by using a boroscopic camera inserted in cracks or in holes drilled in joints, see Figure 3a. From the inspection, it was possible to conclude that the three-leaf walls have external leaves of granite ashlars with a thickness ranging from 0.30 to 0.70 m, while the middle leave is made from loose smaller stones and / or silty soil, see Figure 3b and Figure 4. The combination of heavy rain in Porto, strong winds in the top of the hill where the Cathedral is located, and the open joints in the external masonry face, results in a wet infill even in the summer and the continuous washing out of the infill, see Figure 3c.

(a) (b) (c)

Figure 3. Visual inspection to define the constitution of masonry walls: (a) boroscopic camera, (b) opening up the structure (c) loss of material through central cracks in the openings.

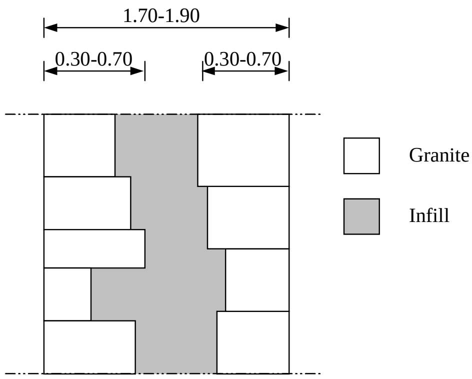
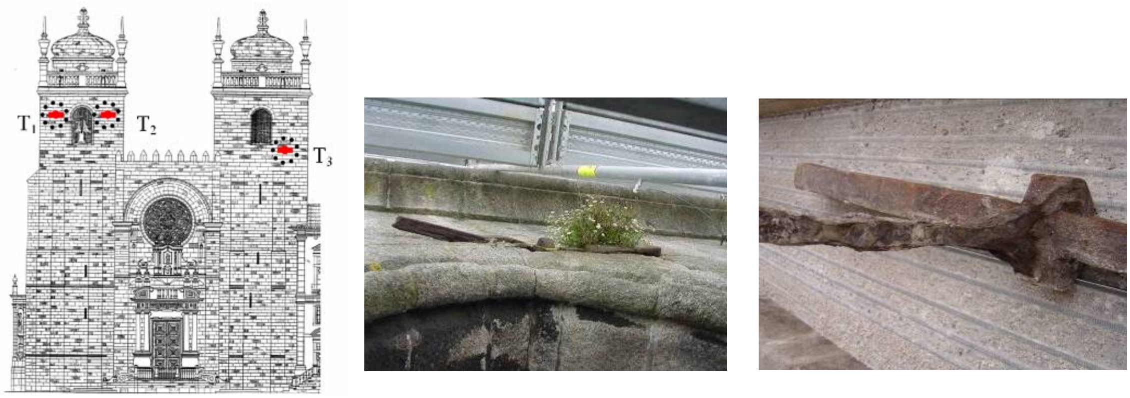


Figure 4. Typical cross section of the masonry walls.

Existing damage

The towers exhibit distributed cracking and significant out-of-plane movements. The existing damage resulted in the past addition of three iron ties (date unknown), see Figure5a. Tie T1 presents a severely deformed anchorage, see Figure 5b, and tie T3 is corroded and broken, see Figure 5c. It is stressed that the separation between the East and West façades of the South tower continued after tie T3 was broken. It is also noted that the masonry walls in the vicinity of the anchorages are also deformed, as expected due to the application of a large point load.



(a) (b) (c)

Figure 5. Ancient tower ties: (a) Deformed anchorage of tie T1 and (b) details of broken tie T3.

The South tower is more damaged than the North tower. Figure 6 exhibits the location of severe cracks and out-of-plumb walls in the South tower. Also the East façade of the South tower presents out-of-plane movements to the exterior. It is noted that the internal walls of this tower are straight, indicating crumbling or desegregation of the walls, with major cracks and voids in the interior, see Figure 7a. The separation between the internal and external leaves of the walls is further confirmed by the longitudinal cracking observed in most of the openings. Figure 7b illustrates such cracking, with a maximum width of some centimetres. Finally, it is noted that the North tower presents severe distributed vertical cracking at the base, see Figure 7c.

This cracking is only visible in the internal (medieval) face, while the external face seems undamaged. Moreover, the very large thickness of the walls is not replicated in the South tower. For these reasons, it is believed that the damage is not recent and the helicoidal staircase with vertical cracking belongs to the structure of an older tower.

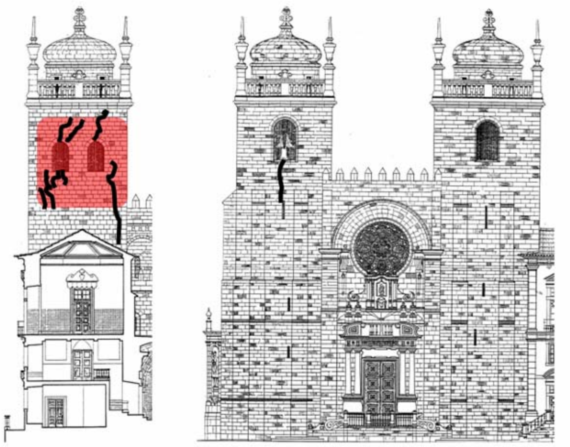
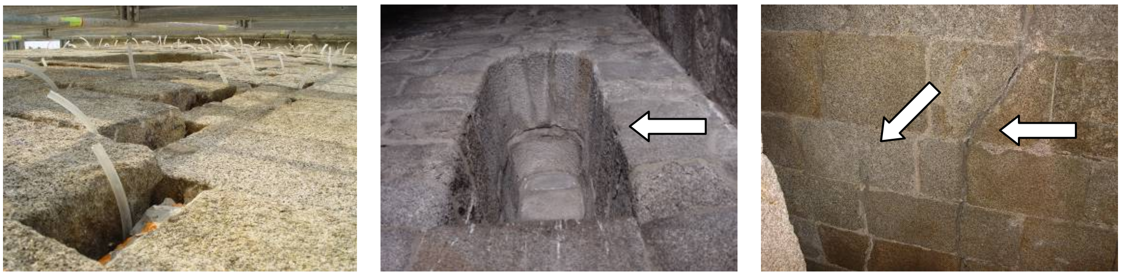


Figure 6. Location of most severe cracks and out-of-plumb walls, in the South view and main façade.



(a) (b) (c)

Figure 7. Details of the cracks in the towers: (a) cracks up to 0.20 m width in the South façade / South tower; (b) typical active cracks parallel to the walls at the openings; (c) vertical cracks at the base of the North tower.

Additionally, also the following damage is noted: (a) Steel structure in the cupolas of the towers with advanced corrosion; (b) Balustrades and pinnacles under deficient stability conditions and with significant movements due to corrosion of dowels and ties; (c) Misconception of the structure supporting the bells and clock in the North tower, see Figure 8.

Figure 8. Deficient structural system to support the bell stone level floor.

Existing damage

As it arises from the history and survey, the towers seem to have been damaged in the past and rebuilt (particularly the South tower). The (re)construction seems to have been carried out under deficient execution conditions, no particular well defined structure and using improvised construction details. In addition, different remedial techniques were already used in the past aiming at correcting and strengthening the towers.

The walls of the towers seem not to possess adequate connection between the external leaves and severe water infiltration in the walls contributed to the existing damage and to the loss of material in the rubble infill. Here it is again stressed that the Cathedral is located at the top of a hill, the masonry joints have lost all mortar and it was found that the rubble infill was wet by the end of the summer. Besides other damage, the most relevant feature is that the North tower is divided in two similar U-shaped parts, from mid-height to the top, with full cracks along the West-East direction (in the other direction, the existing ties kept the tower together), and the South tower is bulging outwards both to South and to East (the existing West-East tie is broken).

The solution adopted for strengthening consists mostly of a steel ring in both towers, aiming at confining the structure along the two orthogonal directions, in the sole location possible, see Figure 9a,b. The rings are made with welded stainless steel plates (class AISI 316L), connected to the towers using long, inclined stainless steel anchorages inside of a cloth duct to prevent generalized injection, see Figure 9c-e. The length of the steel profiles is defined so that the elements can be transported to the location through the existing doors and can be easily assembled in situ, without any further welding.

In the North tower, the ring also aims at providing a support for the stone pavement for the bells. The reason being that the stone columns are very deteriorated and possess presently no structural function and the stone struts have very deficient conception, see Figure 9c. Here it is noted that it was decided not to recuperate the structural function of the columns (e.g. using injection) because the lower level seems to indicate insufficient strength of the inner core, see Figure 9c and Figure 8. The steel ring is made of U profiles (240 × 120 mm and 200 × 100 mm height).

In the South tower, a set of two ties was provided to the ring, because it was possible for aesthetic reasons and they are a witness of the ancient broken tie. The ring must cross the staircase at a selected location because the complex internal structure of the tower does not allow otherwise. Due to the lack of internal stiffening elements, a much more stiff steel frame is needed and the steel ring is made of I profiles (180 × 180 mm). Due to the bulging outwards of the East and South façades, and the severe cracks in the corners, several short ties have been added to the structure to stitch the East and South façades, and two long ties through the core of the South façade have been added to connect the West and East façades, see Figure 9f. Figure 9g presents details of the two types of anchorage plates adopted (circular plates and specially designed crosses).

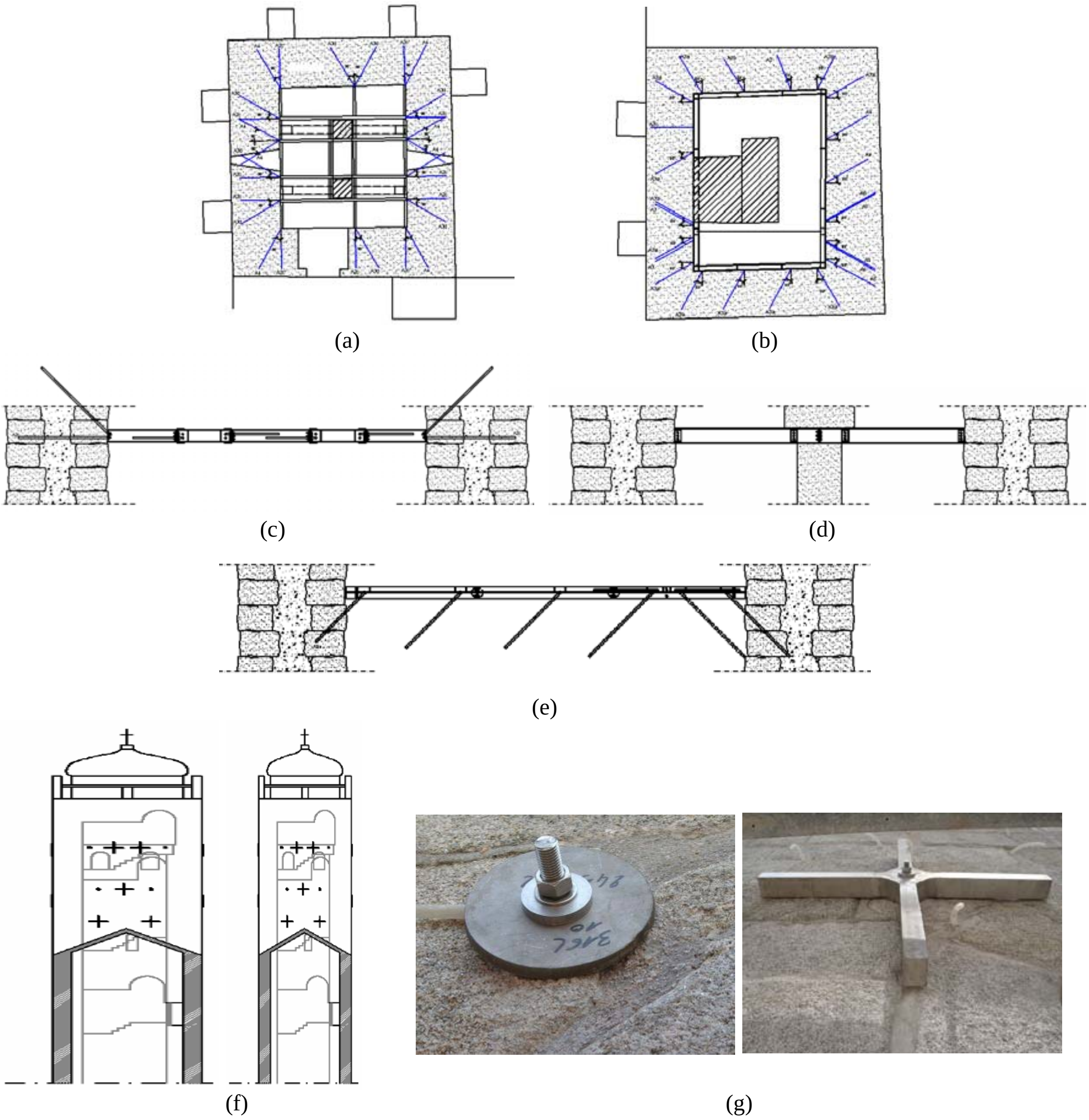
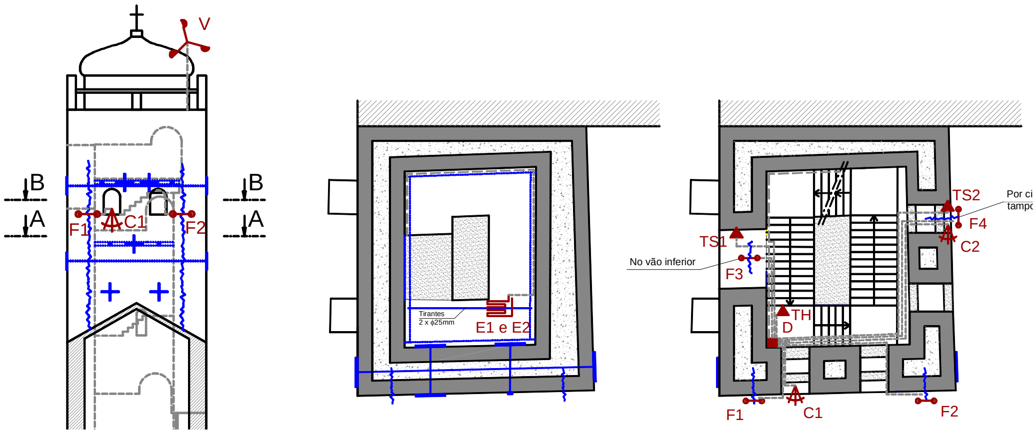


Figure 9. Aspect of the strengthening of the towers using stainless steel rings and long inclined anchorages: (a) plan of the ring for the North tower; (b) plan of the ring for the South tower; (c) North-South section for North tower; (d) West-East section for North tower; (e) typical section for South tower; (f) additional ties placed in the West and South façades of the South tower; (g) details of the anchorage plates.

The replacement of iron dowels and ties by stainless steel was made in pinnacles and balustrades. The large pinnacle in the top of the North tower cupola was totally loose at the time of the works and was jacketed with steel plates at the top and bottom necks. Other works carried out include injection of the main cracks with lime based mortar grout, repointing all joints with selected lime mortars (a traditional mortar for the filling and a more durable lime mortar for the finishing), protect against corrosion (the two ties in the North tower were kept in place) or replacing all existing iron.

Monitoring Plan

Given the cultural importance of the building and the significant damage in the South tower, a monitoring system was planned and installed. The system includes four waterproof crackmeters in the largest cracks, two strain gages for the new ties, two biaxial clinometers to measure the tilting of the tower, as well as temperature, humidity and wind sensors, see Figure 10 and Figure 11. The system includes a datalogger and a GSM interface for remote monitoring, see Figure 12. An example of the measured values, for the crackmeters and temperature sensors, indicate minor variations and the typical seasonal effects, thus confirming the adequacy of the intervention measures.



(a) (b)

Figure 10. Monitoring system in the South tower: (a) South view; (b) plan. Here, F1 to F4 indicate crackmeters, E1 and E2 are vibrating wire extensometers, TS1 and TS2 are temperature sensors, TH is a combined temperature and relative humidity sensor, C1 and C2 are tiltmeters and V is anemometer capable of measuring wind direction and velocity.

(a) (b) (c) (d) (e) (f)

Figure 11. Examples of sensors placed: (a) crackmeter; (b) extensometer; (c) temperature sensor; (d) combined temperature and relative humidity sensor; (e) tiltmeter; (f) anemometer.

Analysis of Saint Vincent Chapel

The Saint Vincent Chapel is located next to the South wing of the Cathedral cloister. During the restoration works of the roof, it was found that the extrados of the chapel vault was filled with rubble resulting from old demolitions see Figure 14. Also, and as usual in several historical constructions, the timber roof was partly supported by the vault, using later added struts. The issue addressed here is the stability of the vault and the convenience of the removal of the infill.

(a) (b) (c)

Figure 12. Data acquisition system and software: (a) datalogger; (b) GSM download; (c) manual data collection.

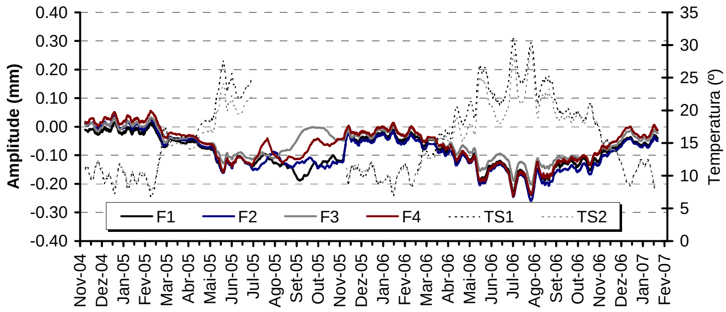


Figure 13. Example of results from crackmeters and temperature.

(a) (b)

Figure 14. Roof of Saint Vincent Chapel: (a) aspect of restoration works; (b) aspect of vault infill with rubble.

Survey

The structure consists of a barrel vault with an approximate thickness of 0.25 m and a span of 6.8 m. On the North side, the cloister acts as a buttress but on the South side no buttresses are present. Even if the South wall (1.70 m) is thicker than the North wall (1.30 m), out-of-plumb movements outwards are clearly visible in the former, up to 1.5% (or 0.10 m at the springer of the vault), see Figure 15a. Nevertheless, as the vault presents only minor cracking, see Figure 15b, it was believed that the vault has been built after the wall deformation. As it will be confirmed next, the vault replaces a previous timber roof at the same level. From inspection pits, see Figure 15c, the geometry of the vault could be determined.

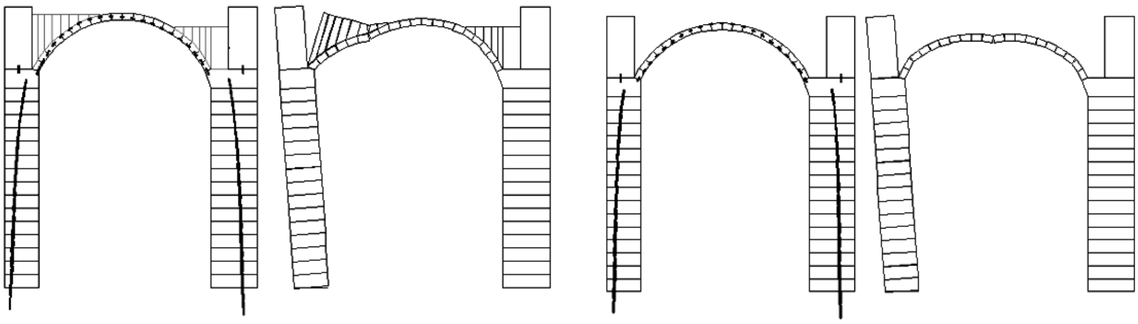
Structural Analysis

A plane model was adopted for the structural analysis of the barrel vault. The analysis was carried out using limit analysis, discretizing the walls and vault as a set of rigid blocks [4]. More complex approaches are available, e.g. [5-7], if necessary for more detailed studies. The assumed material properties include a tensile strength equal to zero, a tangent of the stiffness angle equal to 0.7, zero dilatancy and a compressive strength equal to 6 N/mm^2. The actions included consist only of the self-weight of the structure.

As the objective of the analysis is to evaluate the influence of the infill, a sophisticated representation of the structure is not particularly relevant. Therefore, the influence of the cloister, openings of the walls and ribs of the vault were neglected in order to avoid the need of a three-dimensional model. The numerical results are given in Figure 16, in terms of thrust-lines and collapse mechanisms, both for the model with and without infill. The ultimate load factor increases 45% if the infill is removed, which seems also natural because it was not originally planned for this construction.

(a) (b) (c)

Figure 15. Saint Vincent Chapel: (a) cross-section with 1.5% out-of-plumbness on external (right) wall; (b) aspect of vault intrados; (c) aspect of survey from inspection pit.



(a) (b)

Figure 16. Results of the numerical analysis, in terms of thrust-lines and failure mechanisms: (a) with infill, for a ultimate load factor equal to 6.5; (b) without infill, for a ultimate load factor equal to 9.4.

Remedial measures

The infill was removed but, for safety reasons, it was recommended to accompany this task with topographic measurements, see Figure 17a. The targets were read always at early morning to reduce temperature effects, daily during the process of infill removal (one week) and weekly during one month after load removal. Approximately 35 m^3 (7000 kg) of rubble were removed from the vault and no movements were recorded in the targets. Figure 17b demonstrates that (a) the vault was never conceived to accommodate infill and (b) a timber roof existed at the level of the vault, before the construction of the vault and the new roof at a higher level.

Conclusions

The present paper addresses the works recently carried out at the Cathedral of Porto as a case study. The methodology that governed the complete set of works is addressed and the conservation and repair works are briefly addressed. Three aspects are treated in detail, namely the towers, the Saint Vincent Chapel and the skylight.

(a) (b)

Figure 17. Infill removal: (a) location of topographic targets for monitoring; (b) aspect of the cleaned vault.

The towers exhibit severe global damage including cracking, crushing and separation between leaves and also local damage in the cupolas, pinnacles and balustrades. The global damage seems mostly due to water infiltration, deficient conception of the structure, ancient damage due to lightening and changes in the structures of the towers. For the purpose of increasing the structural performance, a rigid frame of stainless steel profiles and a set of long, inclined anchors have been designed to provide a confining ring. In addition, new ties and stitching of the external leaves were also included when necessary. The local damage is mostly due to corrosion of iron elements, which have been replaced by stainless steel elements or have been protected. The monitoring system confirms the adequacy of the remedial measures.

The chapel exhibits a significant overload due to a rubble infill resulting from previous demolitions and the external wall presents moderate out-of-plane displacements. From the diagnostics, it was possible to safety prescribe the removal of the infill (approximately seven tons). This operation allowed to confirm that the present vault is not contemporary to the walls and the external wall deformation is stabilized.

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